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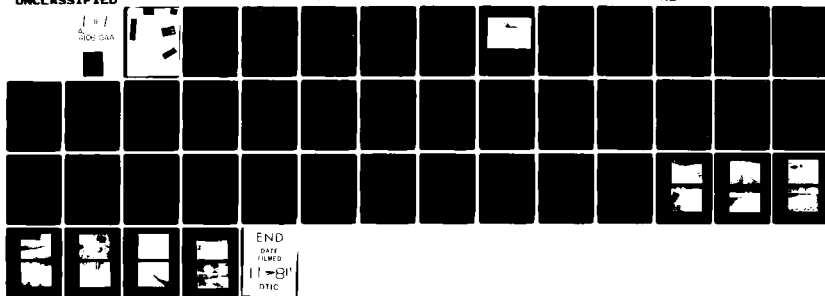
ARMY ENGINEER DISTRICT MEMPHIS TN
NATIONAL DAM SAFETY PROGRAM. BAYLESS TAYLOR DAM (MO 31091), MIS--ETC(U)
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.		

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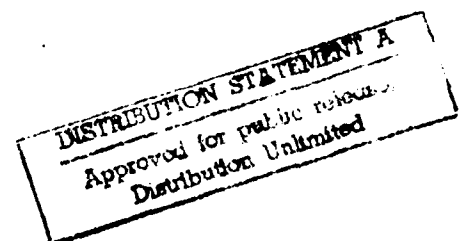
BAYLESS TAYLOR DAM
BOLLINGER COUNTY, MISSOURI
MISSOURI INVENTORY NO. 31091

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM



PREPARED BY: ST. LOUIS DISTRICT CORPS OF ENGINEERS
FOR: GOVERNOR OF MISSOURI

MARCH 1980



PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

NAME OF DAM
STATE LOCATED
COUNTY LOCATED
STREAM
DATE OF INSPECTION

BAYLESS TAYLOR DAM
MISSOURI
BOLLINGER COUNTY
VIRGIN CREEK
17 MAY 1979

Bayless Taylor Dam was inspected by an interdisciplinary team of engineers from the Memphis District, U. S. Army Corps of Engineers. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

The guidelines used in the assessment were furnished by the Department of the Army, Office of the Chief of Engineers and developed with the help of several Federal and state agencies, professional engineering organizations, and private engineers. Based on these guidelines, this dam is classified as a small size dam with a high downstream hazard potential. Failure would threaten the life and property of approximately four families downstream of the dam.

The inspection and evaluation indicate that the spillway does not meet the criteria set forth in the guidelines for a dam having the above mentioned size classification and hazard potential. For its size and hazard category, this dam is required by the guidelines to pass from one-half PMF to PMF. However, considering the high-hazard potential to life and property of approximately four families downstream of the dam, the PMF is considered the appropriate spillway design flood. The PMF is defined as the flood discharge that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region. The emergency spillway for Bayless Taylor Dam will only pass 20 percent of the PMF before the dam embankment is overtopped. Because the spillway will not pass one-half of the PMF without overtopping but will pass the 10-year frequency flood, the dam is classified as "unsafe non-emergency." Also, the spillway will not pass the 100-year flood without overtopping, which is a flood that has a 1 percent chance of being exceeded in any given year. There are no other hydrologic or hydraulic deficiencies.

Other deficiencies visually observed by the inspection team were erosion gullies within the emergency spillway; trees and small bushes on the right downstream abutment; wave wash on the upstream slope of the embankment; brush and tree growth on the upstream slope and at the downstream toe of the embankment; and seepage. Another deficiency found was the lack of seepage and stability analysis records.

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Date	
By	
Signature	
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A	

It is recommended that the owner take action to correct or control the deficiencies described. Corrective works should be in accordance with analyses and design performed by an engineer experienced in design and construction of dams.

Jerry L. Anderson
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SUBMITTED BY: **SIGNED**

Chief, Engineering Division

22 APR 1980

DATE

APPROVED BY: **SIGNED**

Colonel, CE, District Engineer

22 APR 1980

DATE



Overview of Lake

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
BAYLESS TAYLOR DAM - ID NO. 31091

TABLE OF CONTENTS

<u>Paragraph No.</u>	<u>Title</u>	<u>Page No.</u>
SECTION 1 - PROJECT INFORMATION		
1.1	General	1
1.2	Description of Project	1
1.3	Pertinent Data	3
SECTION 2 - ENGINEERING DATA		
2.1	Design	5
2.2	Construction	5
2.3	Operation	6
2.4	Evaluation	6
SECTION 3 - VISUAL INSPECTION		
3.1	Findings	7
3.2	Evaluation	8
SECTION 4 - OPERATIONAL PROCEDURES		
4.1	Procedures	9
4.2	Maintenance of Dam	9
4.3	Maintenance of Operating Facilities	9
4.4	Description of Any Warning System in Effect	9
4.5	Evaluation	9
SECTION 5 - HYDRAULIC/HYDROLOGIC		
5.1	Evaluation of Features	10

TABLE OF CONTENTS (Cont'd)

<u>Paragraph No.</u>	<u>Title</u>	<u>Page No.</u>
SECTION 6 - STRUCTURAL STABILITY		
6.1	Evaluation of Structural Stability	11
SECTION 7 - ASSESSMENT/REMEDIAL MEASURES		
7.1	Dam Assessment	12
7.2	Remedial Measures	12

<u>Plate No.</u>	<u>Title</u>
1	Location Map
2	Vicinity Topography
3	Dam Plan View
4	Centerline Profiles
5	Cross-Section Through Discharge Pipe - 1974 Drawings
6	Cross-Section - Existing Conditions

APPENDICES

- A Hydraulic Computations
- B Geology of Damsite

LIST OF PHOTOGRAPHS

<u>Photo No.</u>	<u>Title</u>
1	Overview of Lake
2	Crest of Dam and Downstream Slope from Right Abutment
3	Overall Crest of Dam from Left Abutment
4	Crest of Dam and Upstream Slope from Right Abutment
5	Wavewash on Upstream Slope
6	Willows in Seepage Area near Emergency Spillway and Left Abutment
7	Emergency Spillway
8	Erosion in Emergency Spillway
9	Seepage at Toe of Dam and Right Abutment
10	Seepage in Willow Area as shown on Photo 6
11	Inlet Structure with Trash Rack
12	Outlet Structure and Stilling Basin
13	Floodplain Below Dam
14	Dwelling Downstream of Dam

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. The National Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of safety inspection of dams throughout the United States. Pursuant to the above, the District Engineer for the St. Louis District, Corps of Engineers, directed that a safety inspection of the Bayless Taylor Dam be made.

b. Purpose of Inspection. The purpose of the inspection was to make an assessment of the general condition of the dam with respect to safety, based upon available data and visual inspection, in order to determine if the dam poses hazards to human life or property.

c. Evaluation Criteria. Criteria used to evaluate the dam were furnished by the Department of the Army, Office of the chief of Engineers, in "Recommended Guidelines for Safety Inspection of Dams." These guidelines were developed with the help of several Federal agencies and many state agencies, professional engineering organizations, and private engineers.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances.

(1) The dam is an earthen embankment built in a relatively narrow valley in the uplands which border the Mississippi Embayment. Topography adjacent to the valley is rolling to steep. Soils in the area are formed of red sandy clays with fragments of dolomite, limestone, and chert. Topography in the vicinity of the dam is shown on Plate 2.

(2) An uncontrolled 14-inch diameter smooth iron pipe with a canopy type inlet sloping through the embankment on a slope of 1V to 5.6H for a horizontal distance of 95 feet is the primary means of discharge. The discharge pipe which is equipped with a trash rack discharges into an earthen stilling basin whose dimensions are 15 feet by 15 feet and whose depth is approximately 4 feet. An emergency spillway is cut in the left abutment. The emergency spillway is a trapezoidal section with an average bottom width of 40 feet and side slopes of approximately 1V to 3.4H on the left and 1V to 6.7H on the right. Also, there is a 10 inch drawdown pipe located at Station 5+15 with an outlet top elevation of 459.7 N.G.V.D.

(3) Pertinent physical data are given in paragraph 1.3 below.

b. Location. The dam is located in the southwestern portion of Bollinger County, Missouri, as shown on Plate 1. The lake formed by the dam as shown on Plate 2 is located on the Zalma, Missouri Quadrangle sheet in Section 10; Township 20 North; Range 9 East.

c. Size Classification. Criteria for determining the size classification of dams and impoundments are presented in the guidelines referenced in paragraph 1.1 c above. Based on these criteria, this dam is in the small size category.

d. Hazard Classification. Guidelines for determining hazard classification are presented in the same guidelines as referenced in paragraph c above. Based on referenced guidelines, this dam is in a High Hazard Classification.

e. Ownership. The dam is owned by Mr. Bayless Taylor, Route 1, Box 91A, Dudley, Missouri.

f. Purpose of Dam. The dam forms a 15-acre recreational and commercial fish production lake.

g. Design and Construction History. The dam was designed by the Soil Conservation Service of the United States Department of Agriculture. Readily available design data was limited to a set of field notes and drawings dated 25 April 1974. The drawings consist of a hydraulic and hydrologic design, a typical embankment cross-section (see Plate 5), field notes for a valley - section and "as built" profile of center line of dam (see Plate 4). Whether or not slope stability and seepage analysis were performed using suitable loading conditions including earthquake forces is unknown.

The dam was constructed in 1975 by George Mouser. The earthen embankment was constructed of red sandy clay which was excavated from the lake area. The soil was transported by a 14 yard scraper and then compacted in lifts by several passings of the scraper and a dozer. The design drawings specified a "core trench" with a 10-foot bottom width at a depth of 3 feet with sides slopes of 1:1 for a length of 500 feet. The actual construction of the "core trench" could not be verified by the inspection team. A typical embankment cross-section from the 1974 design drawings showing the primary features of the discharge system is presented on Plate 5. Based on the inspection survey an average 1V on 2.32 H downstream slope was used instead on the 1V on 2H slope specified. Also, based on the inspection survey, an average of 1V to 3.49H upstream slope was used instead of the 1V on 4H slope specified in the design drawings. An existing valley section was prepared from the 1974 field notes and is presented on Plate 4. Also the "as built" centerline profile of the embankment from the SCS field notes and the present centerline profile from this survey is presented on Plate 4. The design drawings called for an emergency spillway bottom width of 10 feet but it has been enlarged to an average bottom width of 40 feet. Additionally, the design drawings showed a 12 inch smooth iron pipe as the primary discharge structure, but the field inspection revealed that a 14 inch smooth iron pipe had actually been installed in lieu of the 12-inch discharge pipe. The 14-inch smooth iron discharge pipe is located at Station 4+78 with an inlet invert elevation of 477.1 N.G.V.D. and an outlet invert elevation of 460.2 N.G.V.D. As shown in the SCS drawings, the design invert elevations for the 12-inch smooth iron discharge pipe were 476.6 N.G.V.D. for inlet and 462.6 N.G.V.D. for the outlet. There is a 10-inch drawdown pipe located at Station 5+15 with an outlet top elevation of 459.7 N.G.V.D. The inlet side was not located nor observed.

h. Normal Operating Procedures. Normal rainfall, runoff, transpiration and evaporation all combine to maintain a relatively stable water surface elevation. The emergency spillway was reportedly used once in the winter of 1975 when a rainfall of 10 inches in 36 hours occurred. After this event, the spillway was enlarged to its present width. Also the drawdown pipe is used on occasion to lower the lake level to harvest the fish crop.

1.3 PERTINENT DATA

a. Drainage Area - 430 acres (Topographic Quadrangle)

b. Discharge at Damsite.

(1) Discharge can take place through a 14-inch smooth iron pipe with a canopy inlet sloping through the embankment and an emergency spillway.

(2) Estimated experienced maximum flood at damsite - unknown.

c. Elevation (Feet above N.G.V.D.)

- (1) Observed Pool - 477.6
- (2) Normal Pool - 477.1
- (3) Spillway Crest - 479.8
- (4) Maximum Experienced Pool - Unknown
- (5) Top of Dam - Maximum - 483.5
- Minimum - 482.8
- (6) Maximum Pool (PMF) - 484.8
- (7) Invert of Discharge Pipe at Stilling Basin - 460.2
- (8) Streambed at centerline of dam - 461
- (9) Maximum Tailwater - Unknown

d. Reservoir. Length of maximum pool - 2200+ feet.

e. Storage. (Acre - feet)

- (1) Observed Pool - 128
- (2) Normal Pool - 121
- (3) Spillway Crest - 164
- (4) Maximum Experienced Pool - Unknown
- (5) Top of Dam - Maximum - 235
- Minimum - 220
- (6) Maximum Pool (PMF) - 263

f. Reservoir Surface Area (Acres)

- (1) Observed Pool - 15.40
- (2) Normal Pool - 15.02
- (3) Spillway Crest - 17.10
- (4) Maximum Experienced Pool - Unknown
- (5) Top of Dam - Maximum - 22.18
- Minimum - 21.20
- (6) Maximum Pool (PMF) - 24.01

g. Dam

- (1) Type - earth embankment
- (2) Length - 492 + feet
- (3) Height - 22.5 feet Maximum
- (4) Top Width - 12 + feet
- (5) Side Slopes
- (a) Downstream - IV on 2.32H

- (b) Upstream - 1V on 3.49H
- (6) Cutoff - Unknown
- (7) Impervious core - 10-foot wide trench at a depth of 3 feet with side slopes of 1:1 for a length of 500' (Design Drawings)
- (8) Grout curtain - Unknown

h. Diversion and Regulating Tunnel. None.

i. Primary Discharge System.

- (1) Type - An uncontrolled 14-inch diameter smooth iron pipe with a canopy inlet sloping through the embankment on a slope of 1V to 5.6H.
- (2) Horizontal length of 14-inch diameter pipe - 95 feet.
- (3) Invert elevation at entrance - 477.1 N.G.V.D.
- (4) Invert of discharge pipe at stilling basin - 460.2 N.G.V.D.

j. Emergency Spillway

- (1) Type - Uncontrolled earthen
- (2) Width of weir - 40 feet (average bottom width)
- (3) Length of weir - N/A
- (4) Crest elevation - 479.8 N.G.V.D.
- (5) Side Slopes - Left - 1V to 3.4H
Right - 1V to 6.7H

k. Regulating Outlet

- (1) Type - 10 inch smooth iron
- (2) Length of pipe - Unknown
- (3) Invert of pipe in lake - Unknown
- (4) Discharge Invert - 458.9 N.G.V.D.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

The dam was designed by the Soil Conservation Service of the United States Department of Agriculture. Readily available design data were limited to a set of field notes and design drawings dated 25 April 1974. The drawings consist of a hydraulic and hydrologic design, a typical embankment cross-section (see Plate 5), field notes for a valley section and an "as built" profile of the centerline of the dam (see Plate 4). Whether or not slope stability analyses and seepage analyses were performed using suitable loading conditions including earthquake forces is unknown.

2.2 CONSTRUCTION

The dam was constructed in 1975 by George Mouser. The earthen embankment was constructed of red sandy clay which was excavated from the lake area. The soil was transported by a 14 yd. scraper and then compacted in lifts by several passings of the scraper and a dozer. The design drawings specified a "core trench" with a 10-foot bottom width at a depth of 3 feet with side slopes of 1:1 for a length of 500 feet. The actual construction of the "core trench" could not be verified by the inspection team. A typical embankment cross-section from the 1974 design drawings showing the primary features of the discharge system is presented on Plate 5. Based on the inspection survey an average 1V on 2.32 H downstream slope was used instead on the 1V on 2H slope specified. Also based on the inspection survey an average 1V on 3.49H upstream slope was used instead of the 1V on 4H slope specified in the design drawings. An existing valley section was prepared from the 1974 field notes and is presented on Plate 4. Also the "as built" centerline profile of the embankment from the SCS field notes and the present centerline profile from this survey is presented on Plate 4. The design drawings called for an emergency spillway bottom width of 10 feet but it has been enlarged to an average bottom width of 40 feet. Additionally, the design drawings showed a 12 inch smooth iron pipe as the primary discharge structure, but the field inspection revealed that 14-inch smooth iron pipe had actually been installed in lieu of the 12-inch discharge pipe. The 14-inch smooth iron pipe with a canopy type inlet and trash rack is located at Station 4+78 with an inlet invert elevation of 477.1 N.G.V.D. and an outlet invert elevation of 460.2 N.G.V.D. As shown in the SCS drawings, the design invert elevations for the 12-inch smooth iron discharge pipe were 476.6 N.G.V.D. for the inlet and 462.6 N.G.V.D. for the outlet. There is a 10 inch drawdown pipe located at Station 5+15 with an outlet top elevation of 459.7 N.G.V.D. The inlet side was submerged and was not located or observed.

2.3 OPERATION

Normal rainfall, runoff, transpiration and evaporation all combine to maintain a relatively stable water surface elevation. The emergency spillway was reportedly used once in the winter of 1975 when a rainfall of 10 inches in 36 hours occurred. However, the maximum depth of flow that occurred in the emergency spillway is unknown. After this event the spillway was enlarged to its present width. The drawdown pipe is used on occasion to harvest the fish crop.

2.4 EVALUATION

a. Availability. The only engineering data available were mentioned in paragraphs 2.1-2.3 above.

b. Adequacy. The 1974 hydraulic and hydrology information provided by the Soil Conservation Service was adequate for the intended designs of the 25-year and 50-year frequency storms. These design precipitations are much less than the PMF precipitation. Consequently, the design data available were inadequate to assess if the dam could pass the probable maximum flood without overtopping. The surface areas and volume calculations that were used by the Soil Conservation Service agree very closely with those values calculated in this report.

Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

c. Validity. The design drawings appear to be valid except for the differences mentioned in paragraph 2.2 above.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. Visual inspection of Bayless Taylor Dam was performed on 17 May 1979. Personnel making the inspection were employees of the Memphis District, Corps of Engineers, and included a geological engineer, a hydraulic engineer, and a soils engineer. Also, Joe Francine, a neighbor familiar with the construction and operation of the dam, accompanied the inspection team. Specific observations are discussed below.

b. Dam. No detrimental settlement, cracking, slides, or animal burrows were observed in or near the earthen embankment. A typical existing cross-section of the embankment is shown on Plate 6. This section is consistent with the cross-section presented in the Soil Conservation Service 1974 drawings (See Plate 5) except the downstream slope is 1V on 2.32H and the upstream slope is 1V on 3.49H. The top width was measured to be 12 feet. The maintenance of the upstream slope, crest, and downstream slope was satisfactory. (See Photo's 1, 2, 3, 4, 5) About 1½ feet of wave wash occurs near the left abutment (See Photo 5). Minimal willow tree growth occurs on both the upstream slope and on the downstream toe of the embankment (See Photos 4 and 6). Also small tree and bush growth was observed on the right downstream abutment.

Seepage was observed flowing from several locations at the toe of the dam. Most apparent was an area where the downstream toe of the right abutment intersected the valley slope. (See Photo 9) The estimated seepage flow rate was 5 gallons per minute. Seepage also occurred at the toe of the dam near the emergency spillway as evidenced by the willow growth in Photos 6 and 10. The marshy area was located from Station 1+70 to Station 2+70 and was 75 feet downstream from the centerline and was about 60 feet wide. None of the seepage appeared to be piping any material from the embankment or foundation.

c. Appurtenant Structures. An uncontrolled 14 inch diameter smooth iron pipe with a canopy type inlet sloping through the embankment on a slope of 1V to 5.6 H for a horizontal distance of 95 feet is the primary discharge (See Photo 11). The discharge pipe located at Station 4+78 discharges into an earthen stilling basin whose dimensions are 15 feet by 15 feet and whose depth is approximately 4 feet (See Photo 12). The earthen stilling basin appears to be stable with respect to any increasing erosion potential and does not seem to pose any threat to the dam embankment. The respective invert elevations were 477.1 N.G.V.D. for the inlet and 460.2 N.G.V.D. for the outlet. A 10-inch drawdown pipe is located at Station 5+15. The outlet top elevation of this pipe is 459.7 N.G.V.D.

An earthen emergency spillway is cut in the left abutment. The spillway is a trapezoidal section with an average bottom width of 40 feet. The emergency spillway is very well maintained (See Photo 7) except for the extensive erosion that occurs where the flow from the emergency spillway meets the valley floor (See Photo 8). The spillway is approximately 100 feet long from the dam centerline.

d. Reservoir Area. Some wave wash exists on the shoreline but was minimal as compared to the overall shoreline length (see Photos 1 and 3). No excessive erosion or slides were observed.

e. Downstream Channel. The downstream channel is overgrown with trees and brush. However, the flood plain immediately downstream from the embankment is primarily pastureland and is free from excessive brush and tree growth (see Plates 6 and 13).

3.2 EVALUATION

None of the conditions observed are significant enough to indicate a need for immediate remedial action or a serious potential of failure. Visually observed seepage, trees on the upstream and downstream embankment, and erosion gullies in the emergency spillway are deficiencies which, left uncontrolled or uncorrected, could lead to the development of potential problems.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

The primary discharge system and emergency spillway are uncontrolled; therefore, no regulating procedures exist for these structures. The pool is controlled by rainfall, runoff, evaporation, and capacity of the uncontrolled discharge structures. A drawdown pipe is used on occasion to drawdown the lake to harvest the fish crop.

4.2 MAINTENANCE OF DAM

The upstream slope, crest, and downstream slope appear to be satisfactorily maintained. Small brush and willow trees are growing on the upstream slope and at the downstream toe. The primary discharge structure and the emergency spillway are satisfactorily maintained. However, some erosion does exist in the emergency spillway.

4.3 MAINTENANCE OF OPERATING FACILITIES

No operating facilities exist at this dam other than the 10-inch drawdown pipe. The drawdown pipe is reportedly used to lower the lake level in order to harvest a fish crop. However, the method of how the pipe is operated could not be determined. .

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

The inspection team is not aware of any existing warning system for this dam.

4.5 EVALUATION

The maintenance of the dam appears adequate. However, the following recommendations are made:

- a. The bushes and small trees growing on the upstream and downstream embankments and at the downstream toe need to be cut.
- b. The erosion in the emergency spillway needs to be controlled.
- c. Wave wash on the upstream slope needs to be repaired and controlled.
- d. The trees and small bushes on the right downstream abutment need to be cut.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. The available hydraulic and hydrological design appears accurate and adequate for the intended purpose of the initial design.

b. Experience Data. The drainage area was developed using USGS Zalma, Mo. Quadrangle. The lake surface area and storage values were comparable to the 1974 values furnished by the Soil Conservation Service. The spillway and dam layout are made from surveys conducted by the inspecting team. Comparisons were made with the 1974 drawings and the inspection surveys. All relative elevations are comparable with the elevations on the 1974 drawings.

c. Visual Observation

(1) The principal means of discharge is through a 14 inch diameter smooth iron pipe extending for a horizontal distance of 95 feet through the embankment on a slope of 1V to 5.6H.

(2) A trash rack is attached to the canopy inlet to prevent trash blockage of the inlet.

(3) The emergency spillway is well maintained except for the erosion previously noted. The average bottom width of the trapezoidal section is 40 feet.

d. Overtopping Potential. The spillway will safely pass 20 percent of the Probable Maximum Flood (PMF) at a discharge of 670 cfs without overtopping. The Probable Maximum Flood is defined as the flood discharge that may be discharged from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. The PMF will overtop the embankment for a period of 6 hours at a depth of 2.0 feet with a discharge of 4900 cfs. The 1/2 PMF will also overtop the embankment for a period of 4.2 hours at a depth of 1.2 feet with a discharge of 2400 cfs. The 100-year frequency flood will overtop the embankment. For its size and hazard category, this dam is required by the guidelines to pass from one-half PMF to PMF. However, considering the high-hazard potential to life and property of approximately four families downstream of the dam, the PMF is considered the appropriate spillway design. Because the spillway will not pass 1/2 PMF without overtopping but will pass the 10-year frequency flood, the dam is classified as unsafe non-emergency. The data utilized in the preparation of the estimates was various Federal reports, data from field inspection and survey, and output from COE program HEC-1, Dam Safety Version. More specific details will be found in Appendix A.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observations of the dam and appurtenant structures are discussed and evaluated in SECTIONS 3 and 5.

b. Design and Construction Data. The design and construction data were limited to that information discussed in SECTION 2.

c. Operation Records. There have been no known operations which have affected the structural stability of the dam.

d. Post Construction Changes. No post construction changes, other than those referenced in paragraph b above exist which will affect the structural stability of the dam.

e. Seismic Stability. This dam is located in Seismic Zone 2. However, it is located very near the boundary between Seismic Zones 2 and 3. Since this dam is located in Seismic Zone 2 and the proximity of Seismic Zone 3, it is possible that an earthquake could occur of sufficient intensity to cause severe damage or failure of the dam.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. Several items were noted during the visual inspection which should be corrected or monitored. These items are trees and brush on the upstream and downstream embankment face; erosion gullies in the emergency spillway; wave wash on the upstream slope; trees and brush on the right downstream abutment; and observed seepage. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record. Also these analyses should be utilized to detail the corrective actions called for in paragraph 7.2. The Probable Maximum Flood (the spillway design flood) and one-half of the Probable Maximum Flood will both overtop the dam. Because the spillway will not pass the one-half of the PMF without overtopping the dam but will pass the 10-year frequency flood, the dam is classified as "unsafe non-emergency".

b. Adequacy of Information. Due to the lack of engineering design and construction data, the conclusions in this report were based on performance history and external visual conditions. The inspection team considers that these data are sufficient to support the conclusions herein. However, seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available which is considered a deficiency.

c. Urgency. The remedial measures recommended in paragraph 7.2 should be accomplished in the near future. The items recommended in paragraph 7.2a should be pursued on a high-priority basis. The stability and seepage analyses should be given priority by the owner and accomplished without delay in order to determine if corrective measures are necessary. If the safety deficiencies listed in paragraph 7.1 a. are not corrected in a timely manner, they could lead to the development of potential problems.

d. Necessity for Phase II. Based on the results of the Phase I inspection, no Phase II inspection is recommended.

e. Seismic Stability. This dam is located in Seismic Zone 2. However, it is located very near the boundary between Seismic Zones 2 and 3. Since this dam is located in Seismic Zone 2 and the proximity of Seismic Zone 3, it is possible that an earthquake could occur of sufficient intensity to cause severe damage or failure of the dam.

7.2 REMEDIAL MEASURES

a. Alternatives. Spillway size and/or height of dam should be increased to pass the Probable Maximum Flood without overtopping the dam.

b. Seepage and stability analyses should be performed by a professional engineer to assess the safety concerns raised by the seepage present at the downstream toe of the dam. The results of these analyses should be used to apply appropriate corrective measures.

c. O&M Maintenance and Procedures. The following O&M maintenance and procedures are recommended:

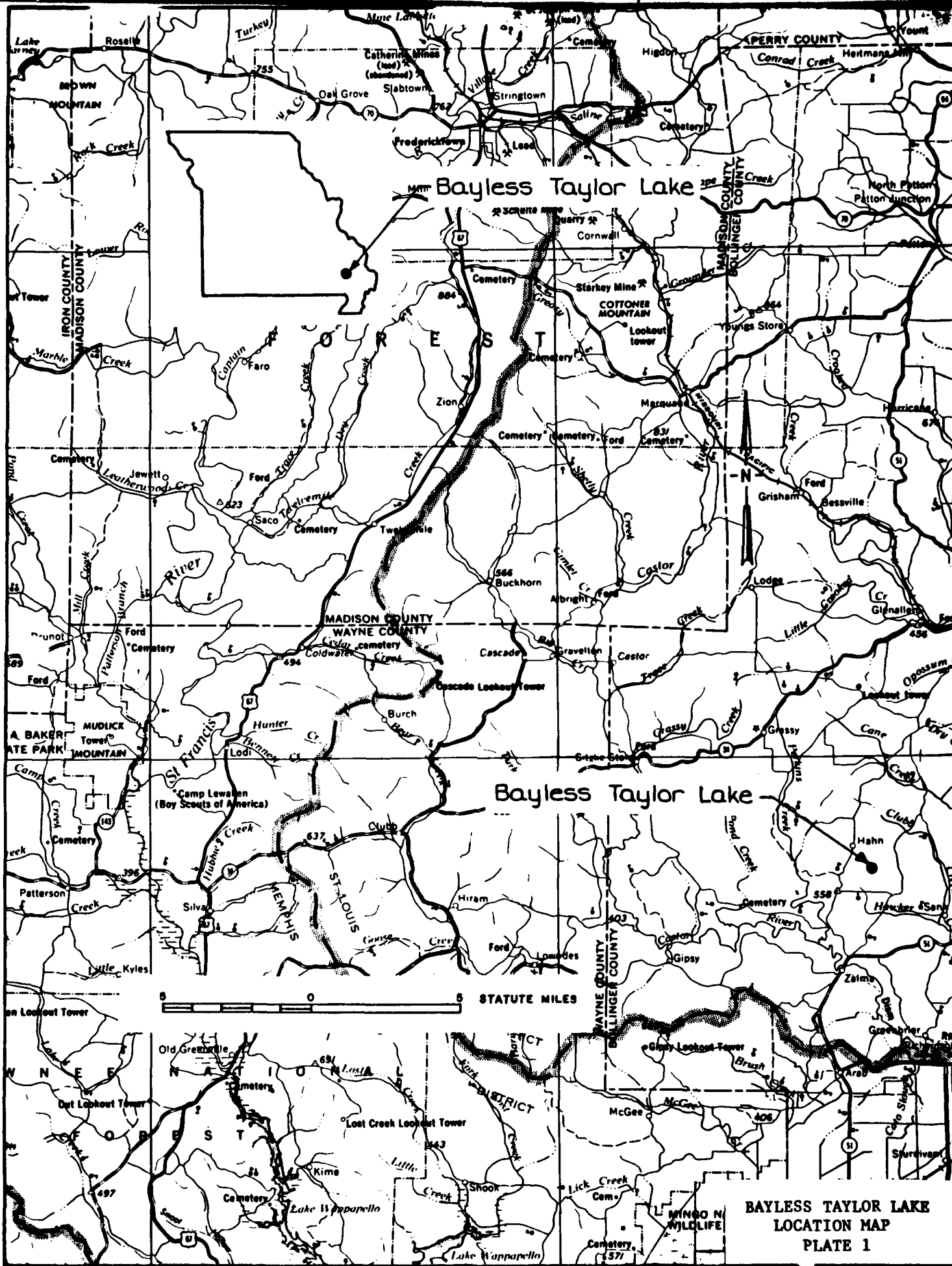
(1) Cut trees and brush on the embankment slopes and right downstream abutment. Care should be taken during removal not to destroy the existing conditions of the embankment.

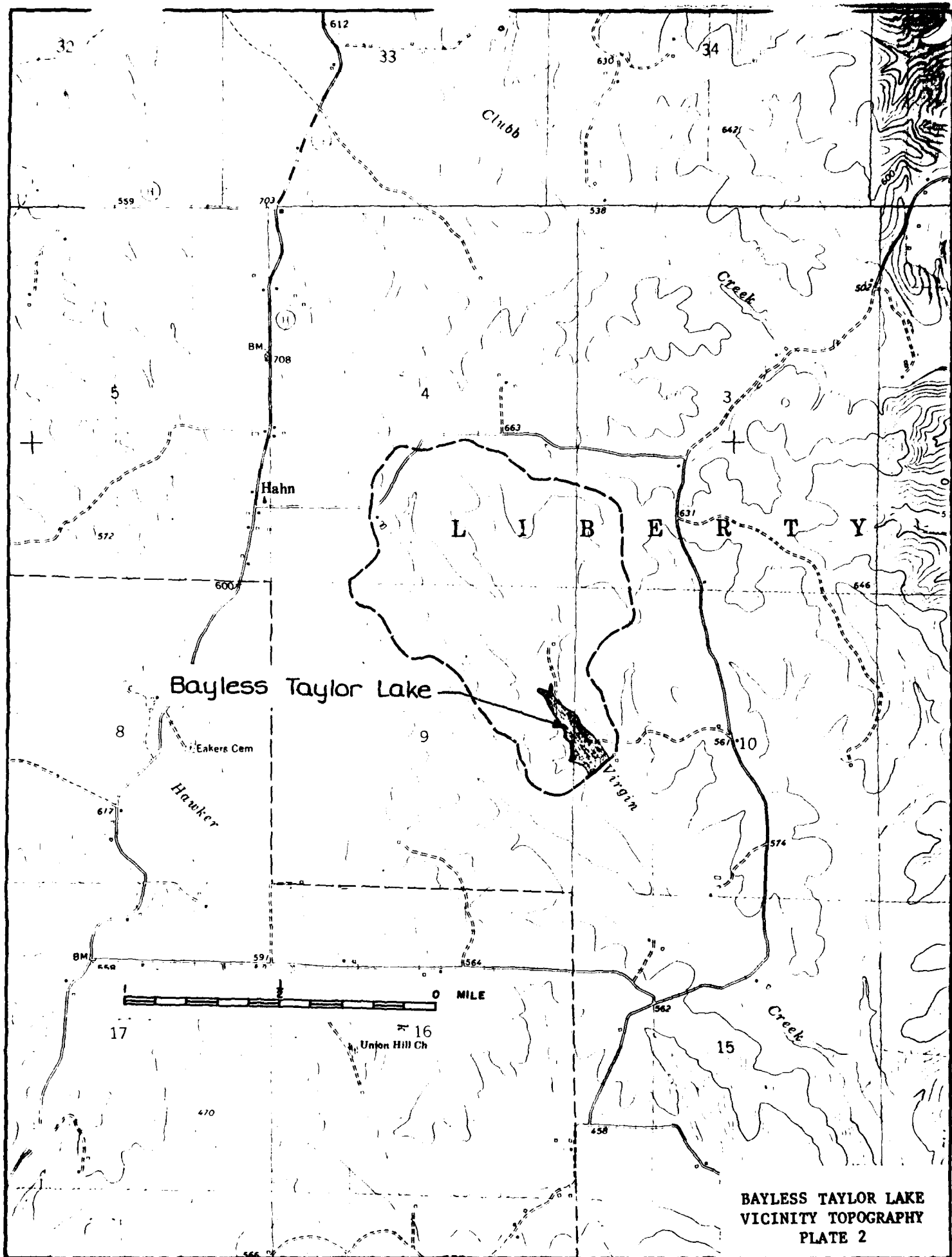
(2) Repair wave wash on upstream embankment slope and provide some type of erosion protection to prevent future occurrences.

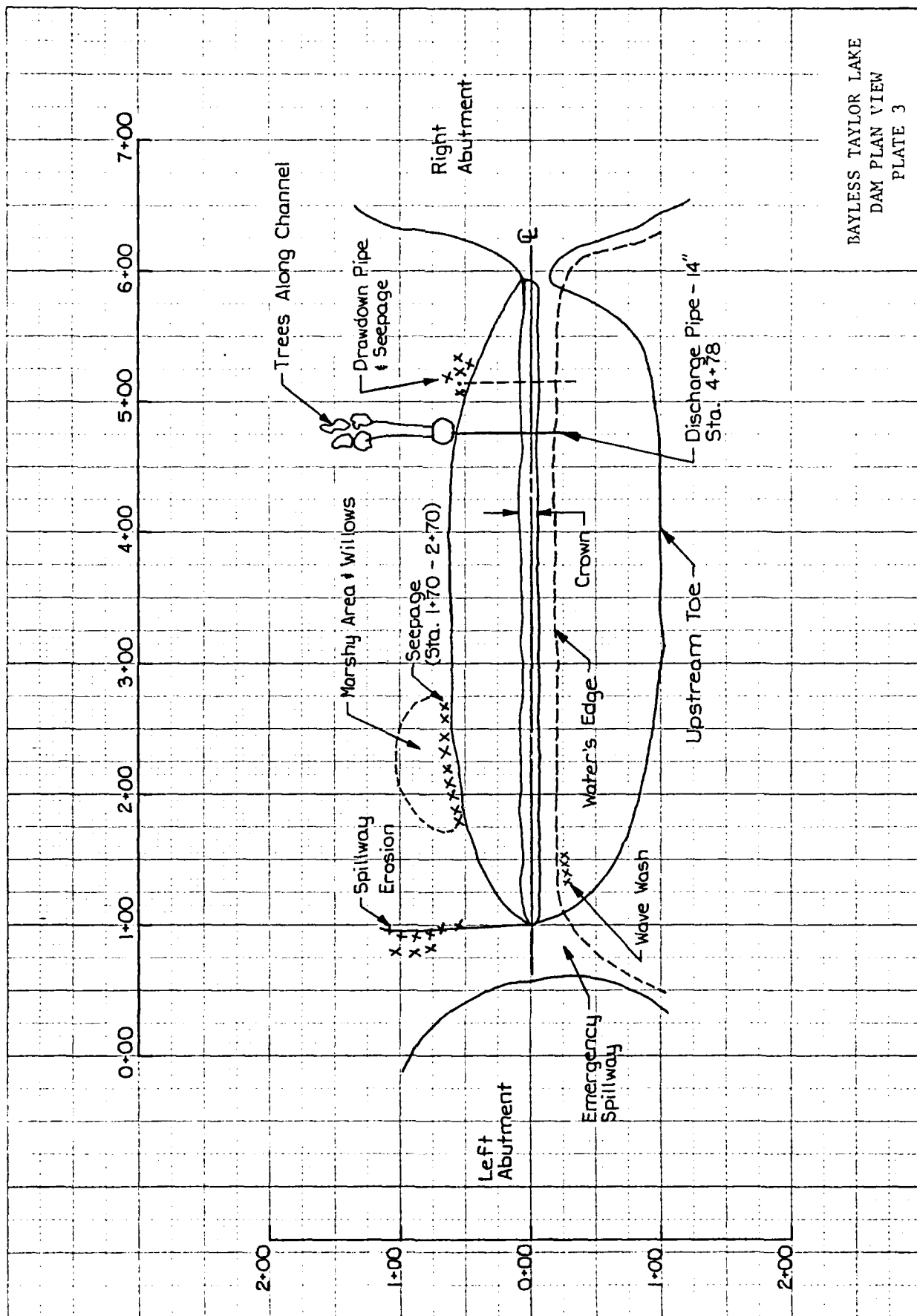
(3) Repair erosion gullies in emergency spillway. Protective measures should be taken to prevent further erosion.

(4) The downstream slope and toe should be closely monitored for seepage and erosion. If seepage quantities and/or erosion observed during monitoring indicate increases or signs of material being piped from the embankment, immediate action should be taken to rectify these conditions.

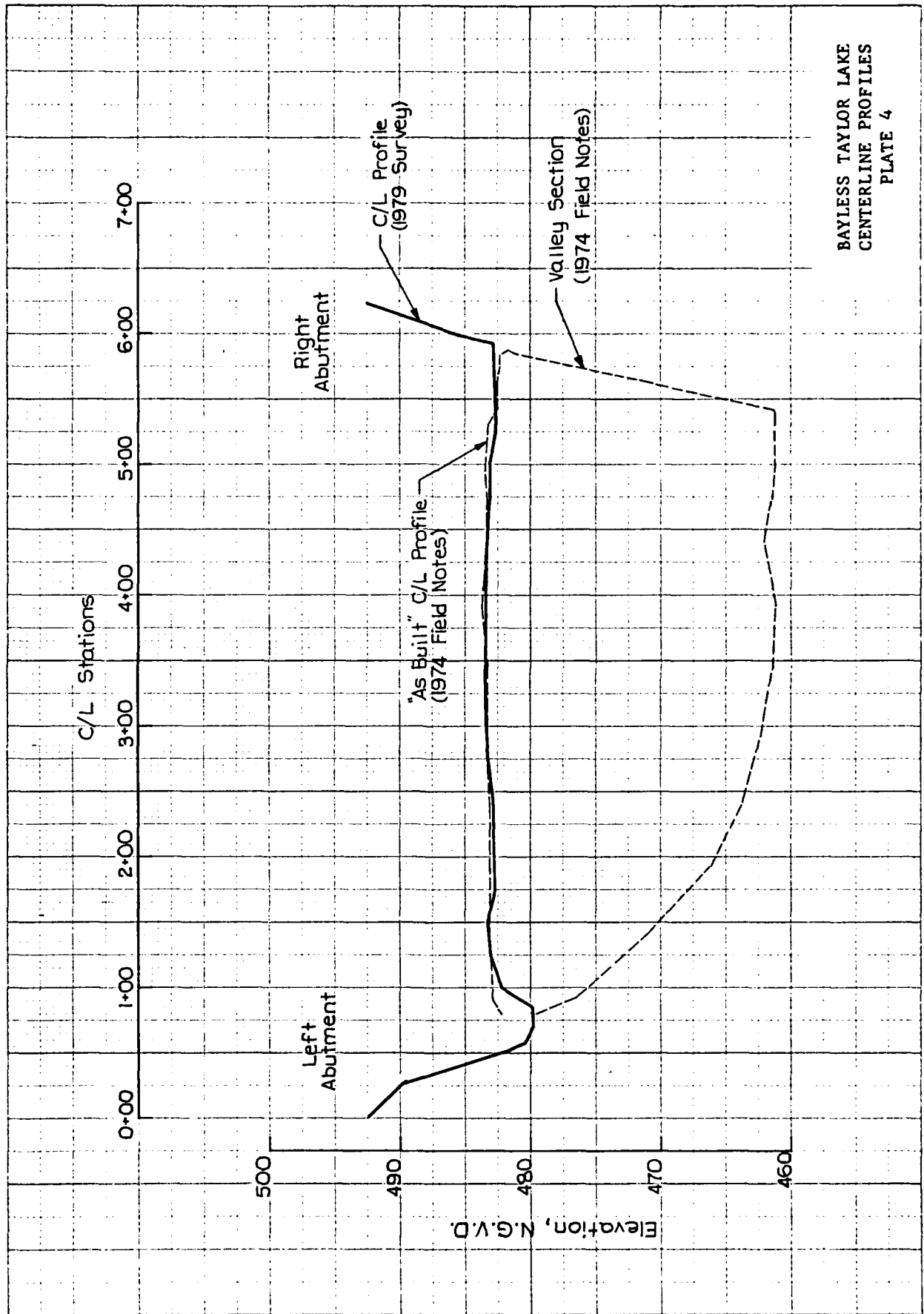
(5) A detailed inspection of the dam should be made periodically by an engineer experienced in design and construction of dams.



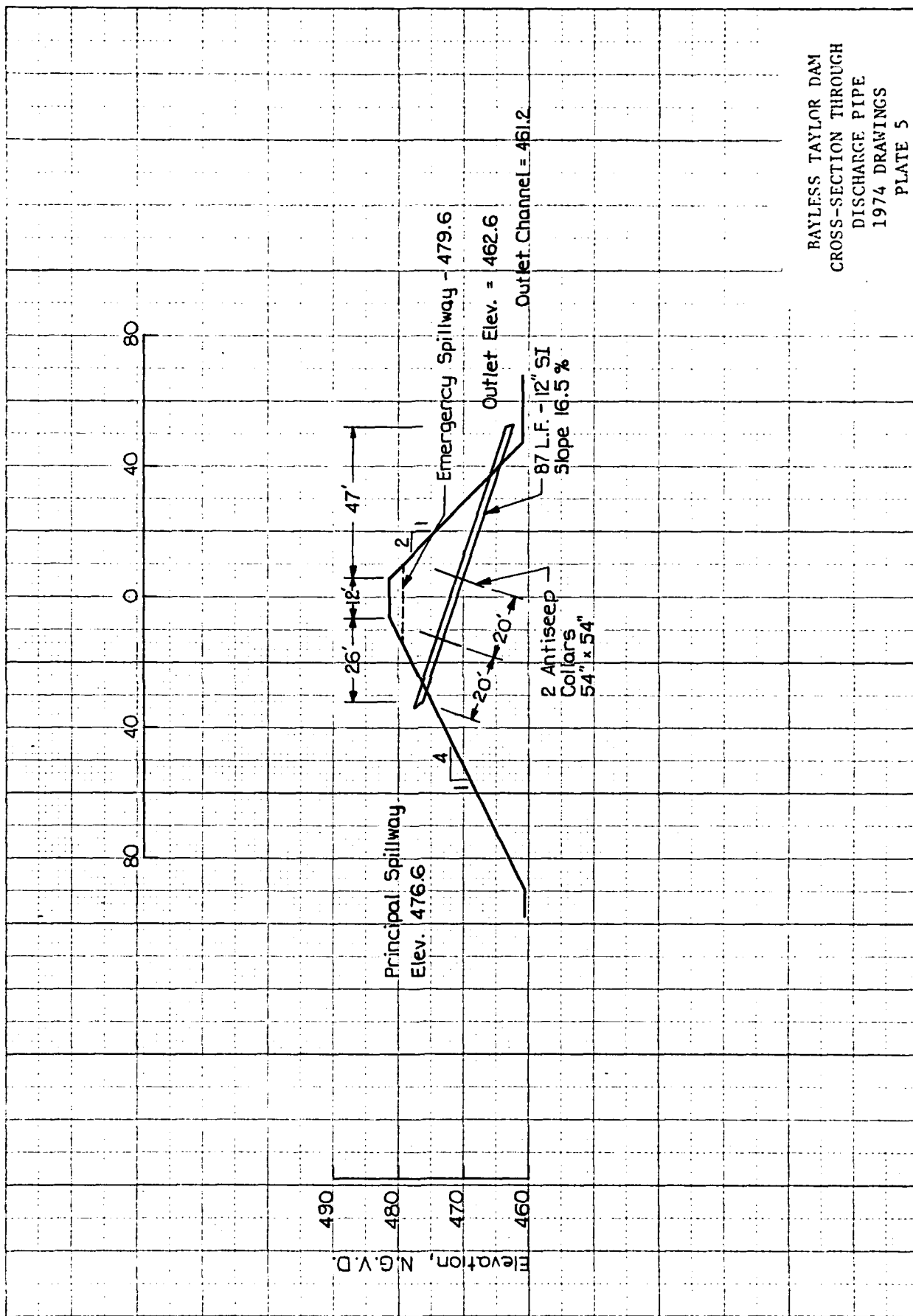




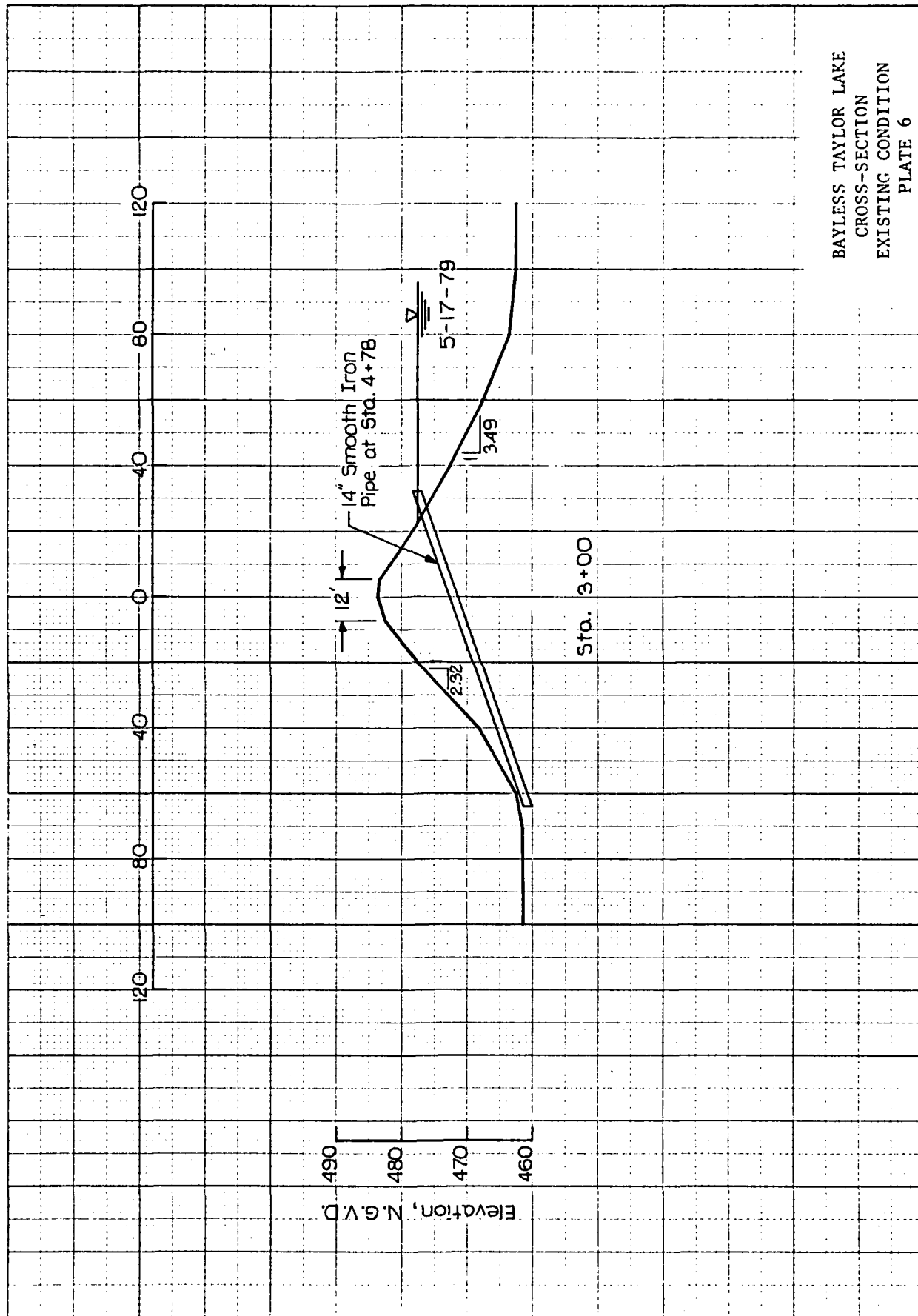
BAYLESS TAYLOR LAKE
DAM PLAN VIEW
PLATE 3



BAYLESS TAYLOR LAKE
CENTERLINE PROFILES
PLATE 4



BAYLESS TAYLOR DAM
CROSS-SECTION THROUGH
DISCHARGE PIPE
1974 DRAWINGS
PLATE 5



APPENDIX A

Hydrology and Hydraulics

1. Narrative. The methods and sources of data were primarily those suggested by the Hydraulics Branch, St. Louis District Corps of Engineers. Specific references and methods will be discussed below. A field inspection survey was made to determine the outlet structures and the topographic characteristics of the dam. HEC-1, Dam Safety Version was used in conjunction with appropriate input parameters to compute inflow hydrographs, determine storage, and route through the structure.

a. Rainfall. The PMF was developed using Hydrometeorological Report No. 33. The "Hop Brook" reduction factor was not used to adjust the rainfall for this study. The distribution of rainfall was developed using the criteria as described by EM 1110-2-1411 (Standard Project Storm).

PMF Rainfall	27 in
PMF Percentages	6 hr - 102
	12 hr - 120
	24 hr - 130
	48 hr - 140

b. Unit Hydrograph Coefficients. The unit hydrograph for the drainage basin was developed using the Snyder Method as outlined in HEC-1 Dam Safety Version. Two methods of determining time of concentration were used for comparison purposes, the Snyder's method and Kirpich method.

The variables used for the appropriate method are listed below.

Snyder's: $t_p = C_t (L L_{ca})^{0.3}$; L and L_{ca} in miles

$L = 7080 \text{ feet} = 1.34 \text{ miles}$

$L_{ca} = 3600 \text{ feet} = 0.68 \text{ miles}$

Stream Slope = 118 ft/mi. = .022 ft/ft

$C_t = .56$

$t_p = .55 \text{ hr}$

$t_c = .63 \text{ hr}$

Kirpich: $t_c = .00013 \left(\frac{L, \text{ft}}{\sqrt{\text{Slope, ft/ft}}} \right)^{.77}$

$t_c = .52 \text{ hr}$

Where

L = length of the main stream channel from the outlet to the divide

L_{ca} = length along the main channel to a point opposite the watershed centroid.

C_t = coefficient used in Snyder's method

t_p = time to peak (hr)

t_c = time of concentration (hr)

Consequently, since the time of concentrations agreed so closely, a value for t_p was chosen to be .50 hr or 30 minutes which necessitated developing a 10 minute unit hydrograph and applying a 48 hr rainfall to develop the inflow hydrographs.

The general soils map of Bollinger County indicates that Bayless Taylor Dam lies in an area where the soil is of the Clarksville Association which is gently sloping to moderately steep soils that have loamy subsoil with a fragipan. This places the area in a Soil Group B. The primary soil cover consists of woods in a fair hydrologic condition which gives a value of CN of 78 for antecedent moisture condition III. Consequently, a value of $C_p = .65$ was chosen as the runoff parameter to be used in Snyder's method.

Listed below are the remaining parameters necessary to develop the unit hydrograph of 10 minute duration.

$$C_p = .646$$

$$\text{Drainage Area} = .672 \text{ sq. mi.}$$

The unit hydrograph ordinates are found in the computer printout.

c. Loss Rates. A loss rate of .5 in. initially and .05 in./hr. was chosen based upon engineering experience.

d. Base Flow and Antecedent Flood Conditions. A base flow of 1 cfs was selected and the routing was started at the low point in the spillway crest of 479.8 N.G.V.D.

e. Hydrograph Routing. HEC-1, Dam Safety Version uses the single routing step of the "Modified Puls" method. Routing through the emergency spillway and over the embankment was accomplished using the non-level dam top option of the HEC-1, Dam Safety Version (see Plate 3) coupled with critical energy consideration of the flow. The routing through the drop inlet structure was obtained considering pipe full conditions with the following assumptions:

Discharge

$$D = 14 \text{ inch smooth}$$

$$L = 95 \text{ feet}$$

$$n = .012$$

head losses:
$$h_{\text{ent}} = \frac{1}{2} \frac{V^2}{2g}$$

$$h_{\text{exit}} = \frac{V^2}{2g}$$

$$h_f = \text{friction loss in 14 inch pipe}$$

$$Q = .6285 A_{14"} \sqrt{2g} H^{1/2}$$

The invert elevation from which to calculate H, height of head, is 460.8 N.G.V.D.

f. Storage. The storage was calculated with the HEC-1, Dam Safety Version with input consisting of elevations and respective surface area which were determined using the USGS Zalma, Mo. Quadrangle.

1. *Journal of the American Medical Association*, 1997; 277: 1039-1043.

RUN DATE: 26 JUN 79
 TIME: 04.02.05

NUN-FEDERAL DAM INSPECTION
DAM #51091
RAYLESS TAYLOR

JOB SPECIFICATION									
NRJ	NRJN	TDAY	IMW	IMIN	MFRC	IPLT	IPRT	NSTAN	
300	10	-0	-0	-0	-0	-0	-0	-0	
		JUPR	NWT	LRPT	TRAC				
		5	-0	-0	-0				

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTU= 9 LRTJOS 1

[illegible]

SIM-AFLA RUNOFF COMPUTATION

PRIMAIRE MAXIMUM PRECIPITATION-48 HR DURATION

ISTAD	ICOMP	TECUM	ITAPP	JPLT	JPRY	INAME	ISTAGE	YALTC
1	0	0	0	0	0	1	0	0

	JUN6	IAMEA	SNAP	HYDROGRAPH DATA				HAI10	ISNOW	ISAME	LOCAL
THYDG	4	47	-	47	47	47	47	-0	-0	-0	-0

SPFF	PMS	PRECIP DATA				R9A	R9B
		R6	R12	R24	R48		
37.00	102.00	120.00	130.00	140.00	-0.	-0.	

LEADPT	SIMKR	DLINKR	WITUL	FMAIN	STPKS	PTTOK	STPTL	CNSTL	ALSMX	PTTTP
0.00	0.00	0.00	1.00	0.00	0.00	1.00	0.50	0.05	0.00	0.00

UNIT HYDROGRAPH DATA
CP = .50 CP = .65 NIA = 0

REFLECTION DATA

STRIDE	1.00	ORCSN2	1.00	R10R2	2.00
GIVEN: SNYDER CP AND IP ARE ICE 3.75 AND RE 2.25 INTERVALS					

UNIT HYDROGRAPH IN FLOOD-PERIOD ORDINATES, LAKE
 524 506 260
 .3 118 145 .67

[illegible]

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PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
 FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
 AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS								
				RATIO 1	RATIO 2	RATIO 3	RATIO 4	RATIO 5	RATIO 6	RATIO 7	RATIO 8	RATIO 9
HYDROGRAPH AT	1	.67	1	.10	.15	.20	.25	.30	.35	.40	.50	1.00
	(1.74)	(492.	738.	984.	1230.	1475.	1721.	1967.	2459.	4918.
ROUTED TO	2	.67	1	.10	.15	.20	.25	.30	.35	.40	.50	1.00
	(1.74)	(13.93)	20.89)	27.85)	34.82)	41.78)	48.74)	55.71)	69.64)	139.27)
	2	.67	1	.10	.15	.20	.25	.30	.35	.40	.50	1.00
	(1.74)	(4.20)	11.48)	18.95)	26.43)	33.90)	41.37)	48.84)	67.99)	138.88)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1

RATIO OF P-4F	ELEVATION STORAGE OUTFLOW	INITIAL VALUE		SPILLWAY CREST		TOP OF DAM		TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
		479.80 164. 23.	479.80 164. 23.	479.80 164. 23.	479.80 164. 23.	482.80 220. 618.	482.80 220. 618.		
MAXIMUM RESERVOIR W.S. ELEV	MAXIMUM STORAGE AC-FT	MAXIMUM DEPTH OVER DAM	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS	TIME OF FAILURE HOURS	TIME OF FAILURE HOURS	TIME OF FAILURE HOURS
.10	481.10	0.	148.	0.	41.67	0.	0.	0.	0.
.15	482.20	0.	405.	0.	40.67	0.	0.	0.	0.
.20	482.90	.10	664.	.50	40.50	0.	0.	0.	0.
.25	483.28	.68	997.	1.33	40.33	0.	0.	0.	0.
.30	483.50	.70	1326.	1.67	40.33	0.	0.	0.	0.
.35	483.64	.84	1592.	2.00	40.33	0.	0.	0.	0.
.40	483.77	.97	1878.	2.83	40.17	0.	0.	0.	0.
.50	483.94	1.19	2401.	4.17	40.17	0.	0.	0.	0.
1.00	484.80	2.00	4904.	6.00	40.17	0.	0.	0.	0.

APPENDIX B

GEOLOGY OF DAMSITE

General Geology. The following geologic information was obtained from a search of the very limited available literature and one field inspection of the dam site and vicinity.

Field investigations conducted at Bayless Taylor Dam revealed a rock outcrop about 10 feet above the crest of the dam along the right abutment (looking down stream). Other than this one outcrop, the entire area was covered with residuum material.

Regional structure of the area is controlled by the Mississippi Embayment, a southerly plunging syncline whose axis is basically outlined by the course of the Mississippi River. The regional dip of the beds is about 1 to 2 degrees toward the Mississippi Embayment. Two major joint systems are present in this area. One system runs northwest to southeast and a second northeast to southwest with vertical fractures. A minor joint system exists in the North-South and East-West directions. The topography and stream patterns of the area are greatly influenced by these joints. Solution zones were found to exist along joints and bedding planes.

Site Description. Bayless Taylor Dam is situated in a relatively narrow valley. The right bank rises on a steep slope and the left bank gently rises to a relatively flat plateau. The valley widens below the dam to about twice its width above the dam. The valley drainage served as a tributary to Virgin Creek prior to dam construction. The embankment abutments and foundation material are predominately the same, consisting of residuum. It is composed of red clay with sand and rock, fragments of limestone, dolomite and chert. The literature indicates the residuum may be between 60 and 200 feet thick. An outcrop was located about 10 feet above the crest of the dam. The rock in the outcrop consist of light gray to light brown, finely crystalline, cherty dolomite with thin beds of sandstone. The outcrop is thought to be Cotter formation of the Ordovician system.

Seepage was observed at the downstream toe of the dam and this problem will be addressed in another section of this report. No other hazardous features such as soft seams, expansive clays or other geologic irregularities were noted. However, the lake is located within the Seismic Risk Zone 2 but very close to the Seismic Risk Zone 3. Because of its locations in Seismic Risk Zone 2 coupled with the steep natural topography, there is a possibility of a sudden landslide into the lake during an earthquake.



PHOTO 1: Overview of Lake



PHOTO 2: Crest of Dam and Downstream Slope from Right Abutment



PHOTO 3: Overall Crest of Dam from Left Abutment



PHOTO 4: Crest of Dam and Upstream Slope from Right Abutment



PHOTO 5: Wavewash on Upstream Slope



PHOTO 6: Willows in Seepage Area near Emergency Spillway and Left Abutment



PHOTO 7: Emergency Spillway



PHOTO 8: Erosion in Emergency Spillway

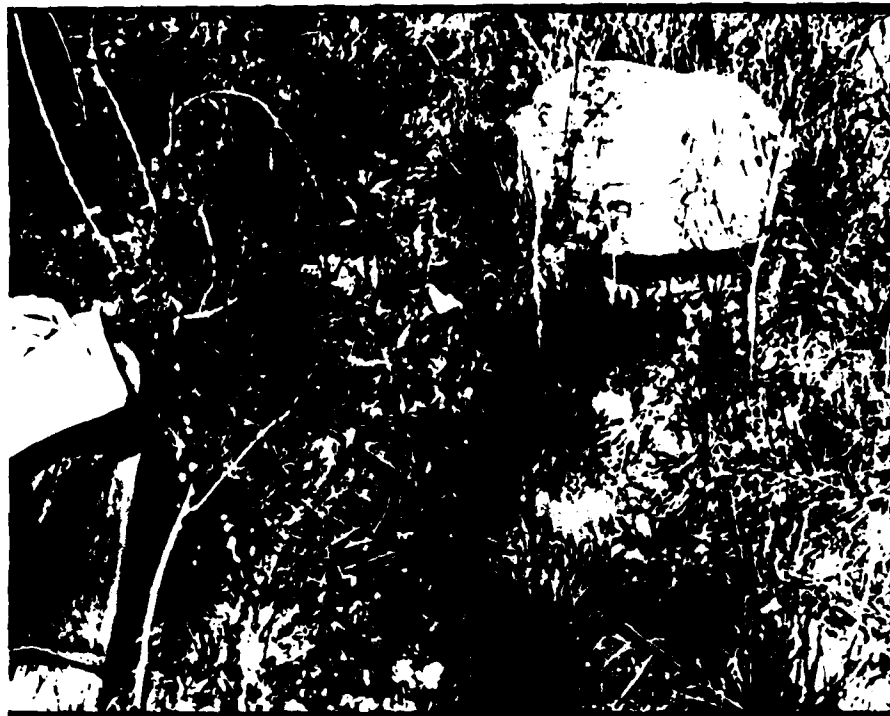


PHOTO 9: Seepage at Toe of Dam and Right Abutment



PHOTO 10: Seepage in Willow Area as shown in PHOTO 6



PHOTO 11: Inlet Structure with Trash Rack



PHOTO 12: Outlet Structure and Stilling Basin



PHOTO 13: Floodplain below Dam



PHOTO 14: Dwelling Downstream of Dam

